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THE PELTON HYDRO-ELECTRIC DEVELOPMENT

J. M. Mullarkey,[1] A. M. ASCE
(Proc. Paper 1823)

SYNOPSIS

This major hydro-electric development is noteworthy for its arch dam, believed to be the thinnest for its height in the United States; for its ski-jump chute spillway; for its fish facilities, including the longest fish ladder ever built, and a downstream regulating dam and reservoir; and for its compact powerhouse and intake design. A description of the project, of the design of the dam and spillway, and of the construction plant and progress is given in this paper.

INTRODUCTION

On December 20, 1957, the No. 1 unit of the Portland General Electric Company's Pelton Hydro-Electric Development first delivered commercial power to the Pacific Northwest. In the first four months of 1958, the remaining two units were completed and the full plant capacity of 120,000 kw is now on the line.

The Pelton development is located on the Deschutes River in the State of Oregon. The site is in Jefferson County some 10 miles N.W. of the town of Madras. In this area, the west bank of the Deschutes River forms the border of the Warm Springs Indian Reservation and the lands on the east bank were withdrawn from private entry by President Theodore Roosevelt and by a law enacted by Congress in 1910 as sites for water power development.

Deschutes River

The Deschutes River rises at elevation 4,000 feet on the east slope of the Cascade Mountains and runs 215 miles in a general northerly course parallel

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1. Project Engr., Ebasco Services Inc., New York, N. Y.

to and close along the base of the Cascades to its junction with the Columbia River some 131 feet above sea level. The Pelton site is approximately 103 river miles above the mouth at elevation 1,400 feet in a canyon eroded 900 to 1,000 feet below the adjacent slopes (Fig. 1). The Deschutes is an ideal power stream having an average flow of 4,200 cfs, a recorded minimum flow of 2,940 cfs and a recorded maximum flow of 13,300 cfs. These flows are taken from a period of record extending back to 1911. Prior to the Pelton development, only one small plant, Bend-1,100 kw, had been constructed on the upper reaches of the Deschutes River.

Brief History and Description of Project

Investigation of the Pelton site was initiated in the 1920's. In 1949 the Portland General Electric Company (through the Northwest Power Supply Company) filed application for a federal license. The subsequent seven years were a period of controversy involving commercial and sports fishery interests, and various state and federal regulatory agencies. These difficulties were finally resolved and in late 1955, final engineering and design was started for the project.

These designs defined the $20 million project (Fig. 2), which consists of:

1. A 204-foot high, thin arch dam spanning 583 feet along the crest between a 28-foot high thrust block on the east abutment and the ogee section of a chute spillway and adjacent gravity cutoff wall on the west bank.
2. The powerhouse which contains three vertical 55,000 hp Francis turbines driving 40,000 kw capability generators and which is located at the toe of the arch dam in the bed of the river.
3. A regulating dam approximately three miles downstream of the arch dam consisting of an 800-foot long, non-overflow, rock-fill section 80-feet high on either side of the 154-foot long concrete center spillway.

The purpose of the regulating dam and reservoir is to restore and maintain the natural flow regime of the river below the project. The concrete spillway also contains the entrances to the 2.8-mile fish ladder which exits into the main reservoir and will enable fish to pass the project during their upstream or downstream migratory period.

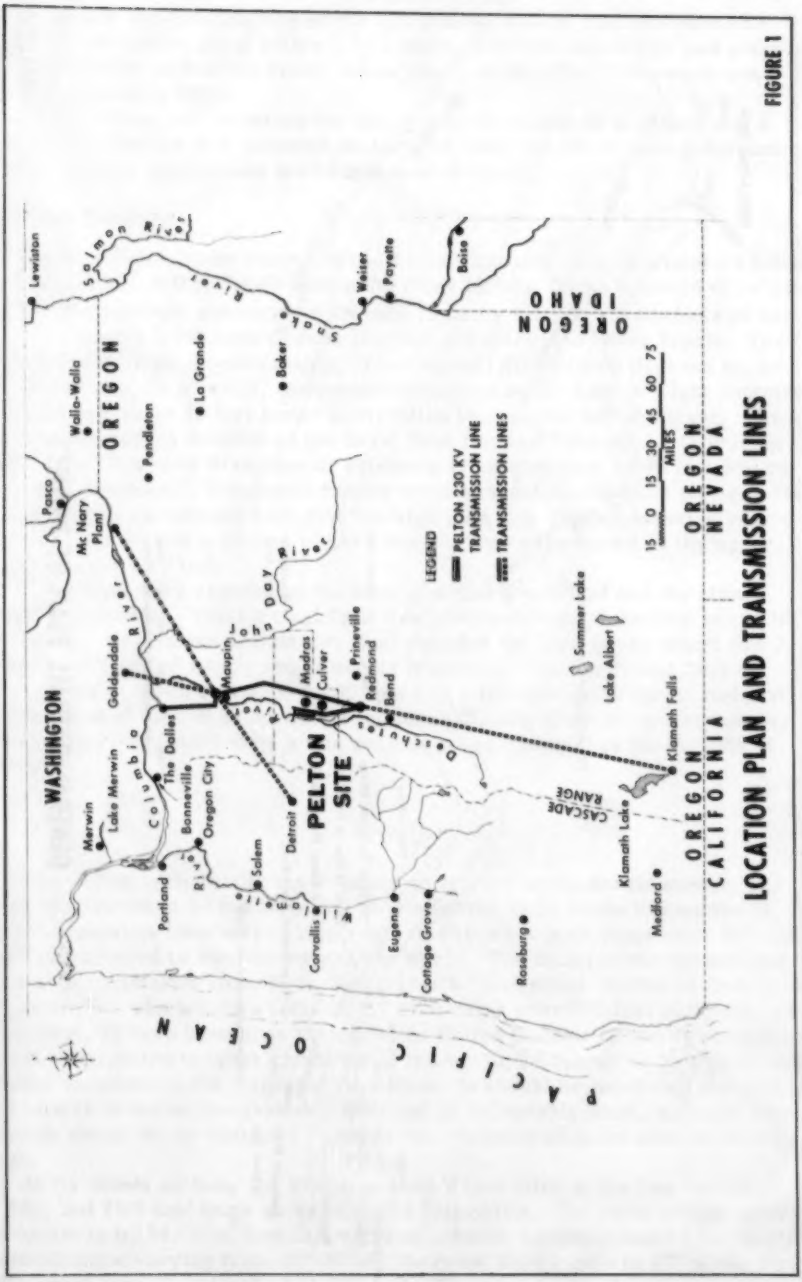
The arch dam impounds a narrow reservoir 7.2 miles long with a total capacity of 37,000 acre-feet. The usable reservoir capacity is approximately 4,100 acre-feet based on a seven-foot drawdown below the normal reservoir level at elevation 1,580. The average annual plant output is estimated at 400,000,000 kwh.

River flows were diverted during construction through a 690-foot long tunnel bypassing the arch dam and powerhouse area and around the regulating dam and spillway by a two-stage cofferdam arrangement.

A 7-1/2 mile, 230-kv transmission line runs south across the plateau to tie the project to the existing regional transmission network.

Preliminary Construction

Construction work started in February 1956, with the construction of a permanent 3-mile access road from U. S. Highway #26 to the dam sites. A portion of this road utilized the abandoned roadbed of the old Oregon Trunk Line railroad. This work was completed in May 1956. The total length of permanent and temporary access roads was more than 7 miles. During the


LOCATION PLAN AND TRANSMISSION LINES
FIGURE 1
LEGEND
PELTON 230 KV TRANSMISSION LINE
TRANSMISSION LINES
15 0 15 30 45 60 75
MILES
PELTON SITE
WASHINGTON
OREGON
IDAHO
NEVADA
CALIFORNIA
PACIFIC OCEAN
Lewiston
Snake River
Weiser
Payette
Boise
Walla-Walla
La Grande
Baker
Pendleton
Pasco
Mc Nary Plant
John Day River
Goldendale
Columbia River
Maupin
Madras
Culver
Prineville
Redmond
Bend
Deschutes River
The Dalles
Lake Merwin
Marwin
Bonneville
Oregon City
Willamette River
Portland
Salem
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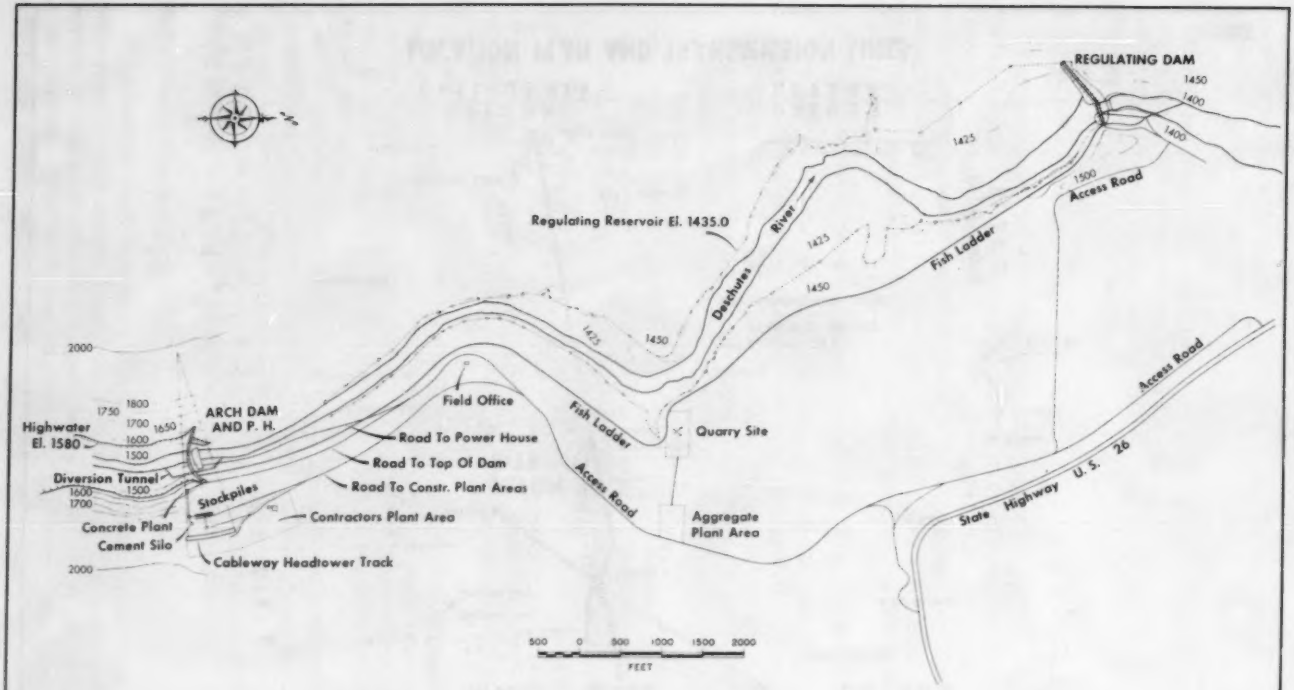
Corvallis
Eugene
Cottage Grove
Roseburg
Medford
Klamath Lake
Klamath Falls
Lake Albert
Summer Lake
CASCADE RANGE



GENERAL OVERALL PLAN OF PROJECT

FIGURE 2

same period, the construction of the 7-1/2 mile, 230-kv transmission line was also underway, along with a 1-1/2-mile, 69-kv tie line which was stepped down to 12.5 kv to provide construction power at the site. This work was completed in July 1956.

Specifications and drawings for the project were issued in March and a unit-price contract was awarded on April 17, 1956, at which time construction of the dam and appertenant structures was started.

Dam Site Geology

The Deschutes River canyon at the Pelton dam site is approximately 1,000 feet deep (el. 2,400 to 1,400) where the river has cut through layers of volcanic rock. The geologic structure of the site is fairly simple and consists of successive, nearly level beds of John Day tuff and Columbia River basalt. The layers are bedded unconformably. The regional dip is three degrees to the Northeast and, as a result, the upper formations in the East or right abutment are exposed some 40 feet lower in elevation than on the left abutment. The base of the dam is founded on the basal John Day tuff formation. Overlying that is the Columbia River basalt extending from elevation 1,400 to 1,560 on the right abutment. The basalt ranges from a massive columnar phase in the lower 45 feet up through a 90-foot brickbat phase, a 25-foot massive vesicular zone and finally a 10-foot clinker phase. Above the basalt is the upper layer of John Day tuff.

Two faults were exposed at the site; one nearly vertical and the other nearly horizontal. The vertical fault lies just upstream of the dam in the left abutment. Excavation across this fault exposed the fault gouge which was 2 inches wide and of nearly impermeable material. The horizontal fault or shearplane at about elevation 1,400 lies 4 to 6 feet below the basalt and just below a baked tuff. It is an undulating plane dipping about three degrees towards the river, marked by a thin zone of gouge material in the vicinity of the dam.

Arch Dam

The Pelton arch dam is the outstanding feature of the development. The dam is believed to be the thinnest, for its height, built in the United States and it compares favorably in this respect with other arch dams over 200 feet high constructed to date throughout the world. The data for the comparison was taken primarily from R. A. Sutherland's "Statistical Review of Dam Construction"[1] which lists a total of 127 arch dams over 200 feet in height. Of this total, 33 have been constructed in the United States. Of the 94 remaining dams constructed in other countries of the world, 10 appear to be thinner than Pelton as shown in the following tabulation. It should be noted that the comparison is based on the available data and is reasonably valid, although the canyon shape factor could not be taken into consideration for lack of information.

At its crown section, the Pelton arch is 8 feet thick at the top, elevation 1,585, and 25.5 feet thick at its base 204 feet below. The ratio of base width to height is 0.124. The dam is a variable-radius, constant-angle type with a central angle varying from 95°-30' at the crest base circle to 77° at the base. The crest base circle radius is 350 feet. The dam might also be termed a dome or double-curvature type of arch (Fig. 3).

Ref. No.(1)	Name	Country	Ht (Ft)	Crest Lth (Ft)	Vol (Cy)	Year Compl	K*
1110	Pelton	U.S.A.	204	583	39,000	1957	.33
1398A	Susqueda	Spain	220	250	6,200	1955	.11
89	Barrea	Italy	205	130	5,500	1952	.24
885A	Moell	Austria	302	557	46,000	1953	.27
1137A	Ponte Racli	Italy	209	391	23,500	1951	.29
1520	Valledi Cadore	Italy	201	125	7,200	1951	.29
651A	Isbert	Spain	203	51	3,000	1955	.29
162A	Bouca	Portugal	213	720	45,850	1955	.30
833	Lois River	Canada	205	990	61,000	1941	.30
868A	Malpasset	France	230	740	52,000	1954	.31
216	Calles	Mexico	220	918	60,000	1928	.32

* K equals Volume in Cubic Yards ÷ (Height in Feet x Crest Length in Feet), and is used only as a convenient measure of the relative thinness.

The dam was constructed in 11 blocks, each approximately 53 feet long, with keyed and grouted radial joints between blocks. Blocks 7, 8 and 9 each contain a section of the three 16-foot diameter penstocks. With the top of the dam only eight feet thick, sufficient width for a roadway was not available, thus requiring a cantilever both upstream and downstream to provide a ten-foot roadway.

There is no intake structure as such. The intake gate and guides are mounted on the upstream face of the dam. The intake hoist structure was erected on a concrete slab which is supported on reinforced concrete beams cantilevering out from the upstream face of the dam at the roadway level. To prevent the downstream migrant fish from being drawn into the turbines the penstock intakes are located as deep as possible, 150 feet below the normal reservoir level. The results of recent experiments show that fingerlings generally travel near the surface of a reservoir(2) and are not likely to sound to such a great depth. To reduce the cost of the trash screens for this very deep intake, the "basket type" screens shown in Fig. 3 were designed. Above the screens, the gate guides were enclosed in structural steel framing covered with galvanized wire mesh fencing and wood panels. Wood panels were used for the top 35 feet. No provisions were made to rake the "basket" trash screens since the Deschutes River carries a very small amount of debris and since the amount of trash collecting on the screens at a depth of 150 feet below the water surface should be negligible. The design of the screen structure is unusual and resulted in a substantial savings in cost over normal design practice.

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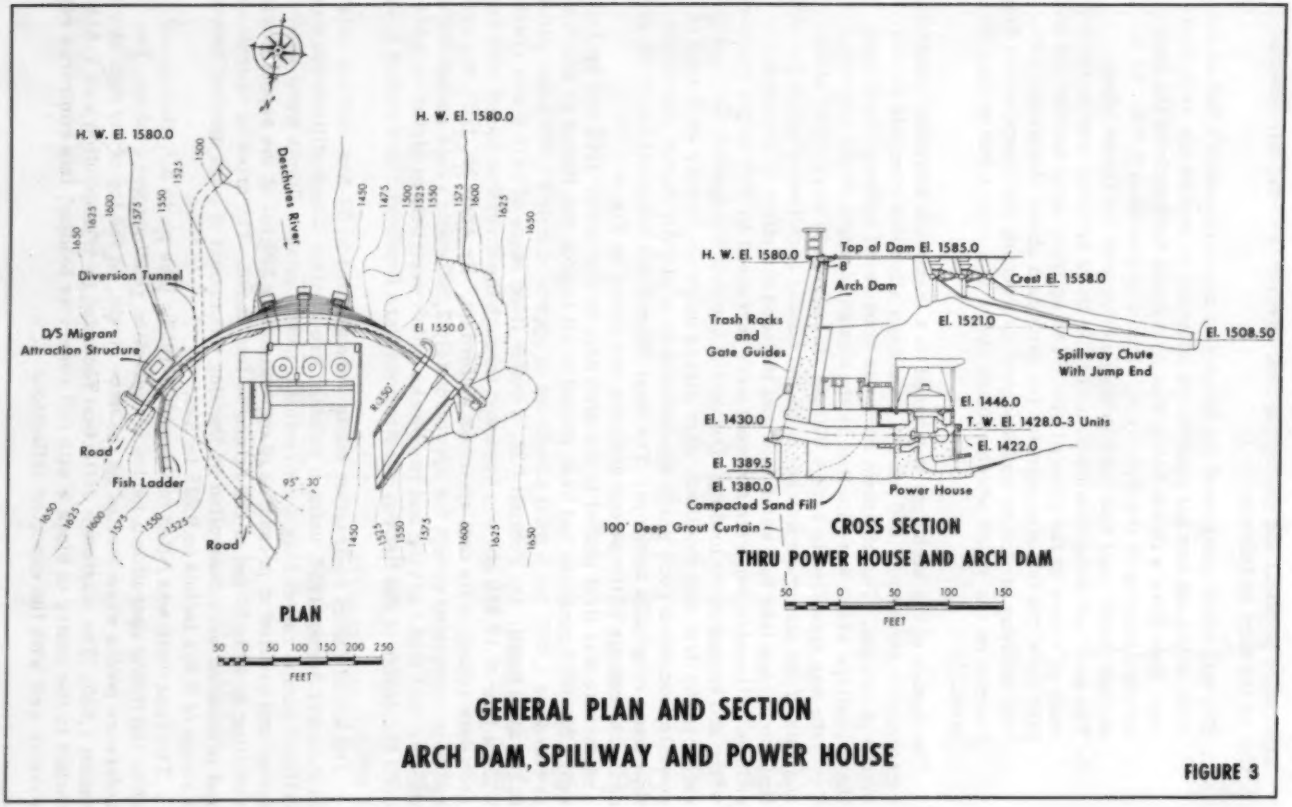


GENERAL PLAN AND SECTION

ARCH DAM, SPILLWAY AND POWER HOUSE

FIGURE 3

The major physical and geological characteristics of the site affected the design of the dam as follows:

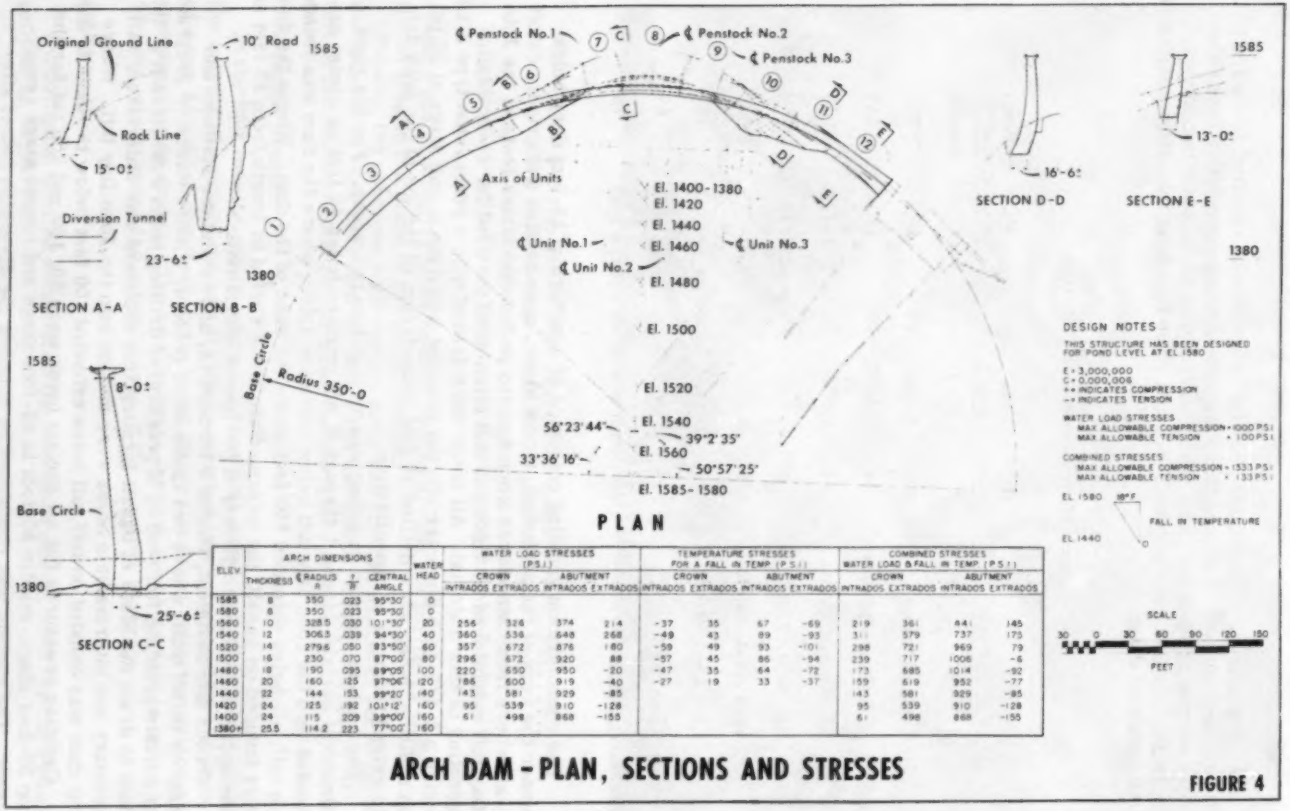
1. The tuff which comprised the foundation above elevation 1,565 on the right abutment was not considered adequate to sustain the arch loadings, therefore a thrust block was interposed to distribute the load through bearing to the underlying clinker at elevation 1,558. An upstream cutoff wall was built in conjunction with the thrust block.
2. The most advantageous location for the chute spillway was on the left bank as shown in the general plan. The spillway ogee section and east pier form the left abutment for the arch dam above elevation 1,540. The spillway structure was designed as a thrust and compression block, founded on the basalt which extends above elevation 1,600 on this abutment.

The design of the arch dam was based on a maximum allowable concrete compressive stress of 1,000 psi and maximum allowable concrete tensile stress of 100 psi. The abutment rock was considered adequate to support these loadings which were limited by the design strength of the concrete. Concrete was specified to have 4,000 psi strength at 91 days. The stress analysis of the structure was based on the elastic arch theory which assumed that the arches take full water load and included the effect of temperature change. Allowable concrete stresses were increased by 33% under combined water and temperature loading. During the construction period, the configuration of the dam was modified, after stress analysis, to take advantage of suitable foundation rock which was uncovered at slightly higher elevations than those originally assumed. The final dimensions and sections of the arch dam, and results of the stress analyses are shown in Fig. 4.

Concrete was first placed in the arch dam in September 1956 and by February 26, 1957 concrete had been placed in all blocks, the lowest of which was at elevation 1,490, for a total placement of approximately 17,000 cubic yards in the arch itself. On February 28, a major flood stage of 1479.8 was reached when a flow of 10,320 cfs was bypassed through the diversion tunnel with the arch dam acting as its own upstream cofferdam. By April 30, 1957, the arch had been completed except for approximately 2,000 cubic yards remaining in the five-foot high top ring and two blocks which were held at elevation 1,560 until the design of the fishway penetrations were finalized. (See photos 1, 2, 3 and 6.)

The keyed radial contraction joints were grouted in 50-foot vertical lifts in July and August 1957, using a patented grout mixer which utilizes two centrifugal pumps rather than a conventional paddle mixer. Joints were water tested and grouted at pressures of approximately 100 psi at the supply header resulting in 40 to 50 psi at the return 50 feet above. The grouting operations and pressures were controlled by limiting the amount of joint opening between a range of 0.015 inches to 0.025 inches.

The reservoir was filled in December 1957. (See photo 8.) During and after the filling operation a series of deflection readings were taken. The reference points were located on the base circle, on the top of the dam at elevation 1,585. The maximum deflection recorded by triangulation was 1-5/16 inches in the center of block 8 with full reservoir loading; this compares reasonably well with the expected deflection.



ELEV	ARCH DIMENSIONS				WATER HEAD	WATER LOAD STRESSES (P.S.I.)				TEMPERATURE STRESSES FOR A FALL IN TEMP. (P.S.I.)				COMBINED STRESSES WATER LOAD & FALL IN TEMP. (P.S.I.)			
	THICKNESS	RADIUS R	t/R	CENTRAL ANGLE		CROWN		ABUTMENT		CROWN		ABUTMENT		CROWN		ABUTMENT	
						INTRADOS	EXTRADOS	INTRADOS	EXTRADOS	INTRADOS	EXTRADOS	INTRADOS	EXTRADOS	INTRADOS	EXTRADOS	INTRADOS	EXTRADOS
1585	8	350	.023	95°30'	0												
1580	8	350	.023	95°30'	0												
1560	10	328.5	.030	101°30'	20	256	326	374	214	-37	35	67	-69	219	361	441	145
1540	12	306.3	.039	94°30'	40	360	536	648	268	-49	43	89	-93	311	579	737	175
1520	14	279.6	.050	83°50'	60	357	672	876	180	-59	49	93	-101	298	721	969	79
1500	16	230	.070	87°00'	80	296	672	920	88	-57	45	86	-94	239	717	1006	-6
1480	18	190	.095	89°05'	100	220	650	950	-20	-47	35	64	-72	173	685	1014	-92
1460	20	160	.125	97°06'	120	186	600	919	-40	-27	19	33	-37	159	619	952	-77
1440	22	144	.153	99°20'	140	143	581	929	-85					143	581	929	-85
1420	24	125	.192	101°12'	160	95	539	910	-128					95	539	910	-128
1400	24	115	.209	99°00'	160	61	498	868	-155					61	498	868	-155
1380+	25.5	114.2	.223	77°00'	160												

ARCH DAM - PLAN, SECTIONS AND STRESSES

FIGURE 4

Foundation Grouting

A very extensive foundation grouting program was required to insure a tight reservoir and to assure that there would be no percolation under the very narrow base of the dam. The program was divided into two phases, consolidation and cutoff curtain grouting. The following tabulation summarizes the pertinent data:

Arch Dam Foundation Grouting

Type	No. Holes	Total Length (Ft.)	Spacing - Ft. Max	Min	Total Grout Take (Sacks)
Consolidation	333	4,743	10	10	703
D/S Consolidation	34	1,509	10	10	487
Rt. Abut. Curtain	84	6,030	10	1-1/4	2,020
Rt. Abut. Rim	4	420	Fanned Vert		430
Left Abut. Curtain	76	8,814	10	1-1/4	3,802
Left Abut. Rim	4	850	Fanned Vert		147
Tunnel Plug	14	560	5	5	69
Contact	One outlet every 40 square feet				76

The consolidation grouting consisted of a pattern of 15-foot deep holes under the base of the arch dam, thrust block, ogee section and gravity cutoff wall where these structures were founded on basalt. Water tests of the John Day tuff established its tightness and eliminated the need for consolidation grouting of this material. All holes were drilled with a percussion type drill using a 2-1/2-inch diameter bit; the grouting was done in one stage at 30 psi. In addition, holes were drilled 15 feet downstream of the toe of the arch dam to extend the area of consolidation.

The curtain and rim grouting was divided into two areas. For the right abutment curtain, blocks 1 through 9, all primary holes 40 feet on center extended into the John Day tuff below elevation 1,400; where the dam was founded on tuff, all holes extended 100 feet below the base of the dam. Where the dam was founded on basalt, the intermediate holes varied in length from 35 feet at the upper elevations to 75 feet at the lowest elevations.

On the left abutment, blocks 9 through 12, and behind the spillway and gravity cutoff wall, where two faults were present, a more intensive program of curtain grouting followed by a program of drilling holes downstream of the dam to drain the block of basalt forming this abutment was undertaken. All primary and intermediate holes were drilled into the John Day tuff. Where the dam was founded on tuff, all holes extended 100 feet below the base of dam.

Grouting pressures for all curtain holes were 50, 100 and 200 psi for the top 30-foot stage, middle 30-foot to 65-foot stage and lowest stage respectively. The curtain was completed by plugging the diversion tunnel after

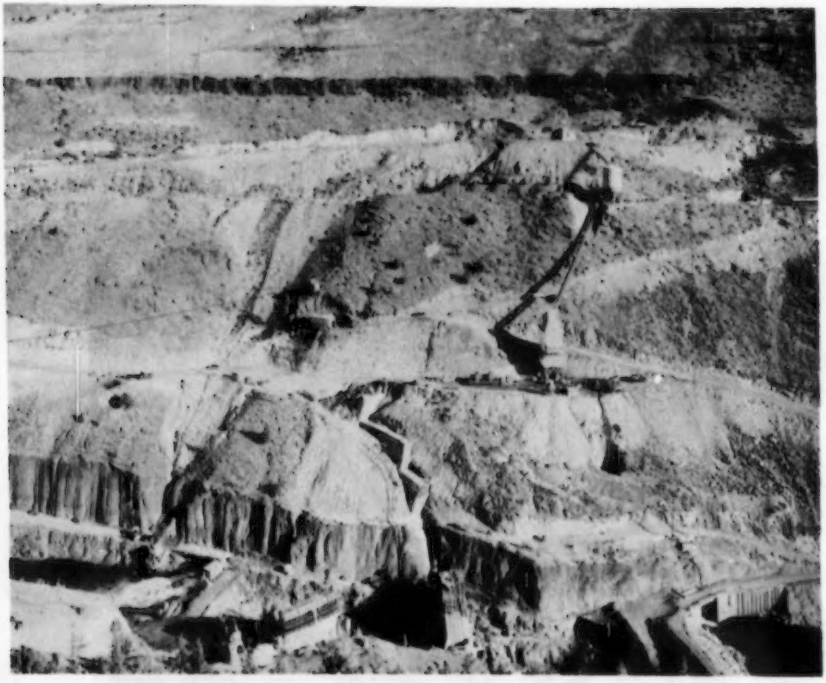


PHOTO #1 - VIEW LOOKING EAST - JAN. 1957
Dam under construction; Diversion Tunnel Portals
& Cofferdams; Cableway; Aggregate Stockpiles;
Concrete Plant; Network of Temporary Access Roads

closure and grouting 14 holes equally spaced around the periphery of the tunnel plug in the plane of the grout curtain.

Finally, the concrete rock contact area was grouted wherever the slope of the foundation rock exceeded 40° above the horizontal, a length of some 300 feet of the 960-foot total length along the periphery of the dam.

Foundation Seepage

In the four months period since the reservoir was filled (December 1957), the amount of seepage through the abutments has been observed by a system of some 46 drilled abutment drain holes, three tile drain collector systems and powerhouse drains. The total seepage through the abutments is only 60 gpm, which shows the effectiveness of the grouting program.

Spillway

Hydrological studies established the maximum design flood for the Pelton project as 30,000 cfs.

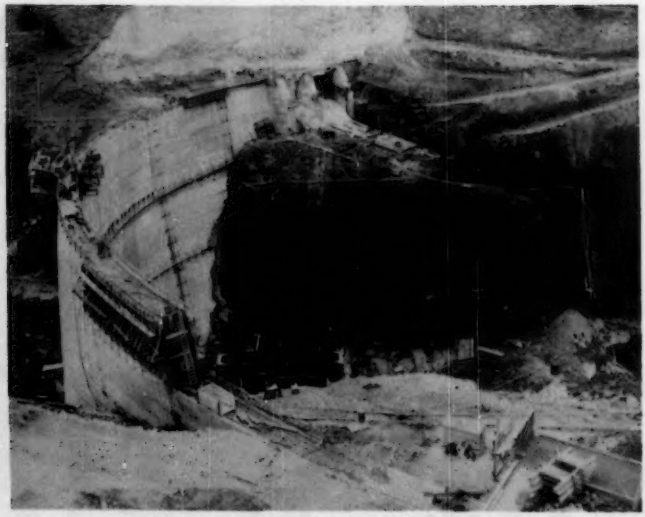


PHOTO #2 - VIEW LOOKING TOWARD SPILLWAY - MAY 1957
Arch dam 90% complete - Spillway chute under construction - Cantilever forms for intake deck

PHOTO #3 - VIEW OF OVERALL PROJECT - MAY 1957
Taken from rimrock on east side of canyon

PHOTO #6 - UPSTREAM FACE OF ARCH DAM
Intake screens partly completed - Diversion portal in foreground - Conc. plant at right - NOV. 1957

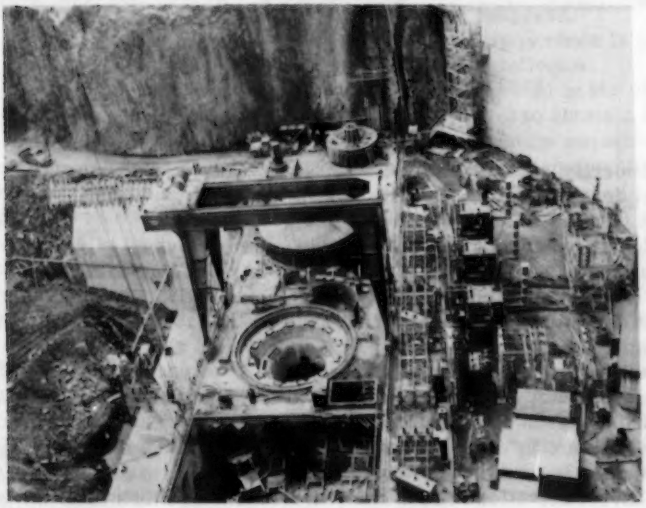


PHOTO #7 - POWERHOUSE CONSTRUCTION
No. 3 spiral case in foreground - NOV. 1957

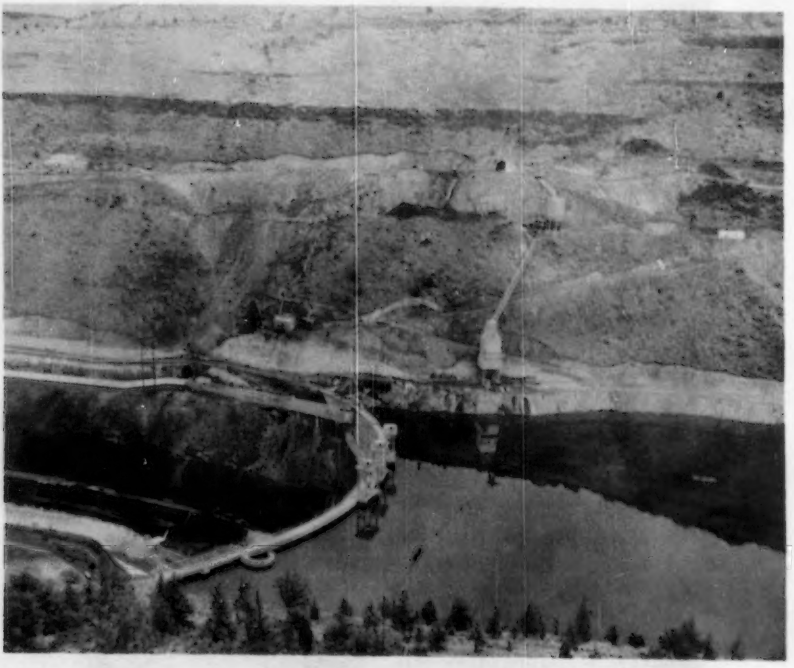


PHOTO #8 - GENERAL VIEW - JAN. 1958
Reservoir full - Looking East

Engineering studies of alternate project layouts established the most economical development as one with a chute spillway, and this type of design was adopted (Fig. 3). Flood discharges are controlled by two 34-foot-wide by 22-foot-high taintor gates installed above the ogee crest at elevation 1558. An approach channel to the ogee section is provided by excavation to elevation 1550. The concrete-lined chute portion of the spillway extends approximately 200 feet downstream from the crest and terminates at the edge of the basalt cliff, approximately elevation 1,510, some 80 feet above tail water. The walls of the spillway chute contract at an angle of 4^0 on each side to reduce excavation and concrete, and then the uphill wall diverges again, from a point where the standing waves reflect, at an angle of 8-1/2 degrees. This expansion decreases the unit "q" at the lip of the chute and reduces the height of uphill wall required to contain the flood discharge.

The invert elevations of the chute are constant across its width on any line perpendicular to the extended centerline of the intermediate pier. As shown on the plan view in Fig. 3, the east end of the chute is 12.5 feet higher than the west extremity. This "superelevation" tends to concentrate and roll the discharge into a compact jet and direct the trajectory downstream. The trajectory is improved by the addition of a concrete fillet at the spillway chute lip which rises 3.5 feet in 13 feet and provides a "ski-jump" effect. This

fillet, which is askew to the flow, provides a downstream velocity component which contributes appreciably to the compactness of the jet. The intermediate training wall also concentrates the flow, thereby adding to the effects obtained by the configuration of the end of the chute.

The design and layout of the spillway chute was based on the theories of high velocity flow and took into account the principles of wave mechanics. A major factor involved in the spillway design was the requirement that downstream migrant fish (fingerlings) be protected during periods of spill. Based on recent studies, which demonstrate that fingerlings can survive a free fall from any height into a pool of water,[2] the chute was designed with a ski-jump deflector and an intermediate training wall to provide a free-falling jet at all flows from 500 cfs through 30,000 cfs.

Spillway Model Study

The spillway arrangement, as designed, was simulated in a hydraulic model which was constructed to a scale of 1 to 60. The dam, powerhouse, spillway, 400 feet of reservoir topography and 700 feet of downstream river topography were included in the model. An extensive testing program in the model covered the performance of the approach channel, the chute and its walls, the ski-jump deflector, and determined the amount of river bed erosion, tailrace disturbance and the spillway discharge calibration.

The major modifications to the spillway layout as a result of the model study were:

1. The addition of the curved approach-channel guide wall at the pier adjacent to the dam which redirected the entering flow so that the discharge through each of the gates was essentially balanced.
2. The final configuration of the lower end of the spillway chute in plan and elevation and the final dimensions of the ski-jump deflector.
3. The location and height of the intermediate training wall in the chute which concentrated low discharges (500 to 3,000 cfs) to obtain a free-falling jet which cleared the cliff face. A gate operation sequence which is required in conjunction with the training wall was also developed.

After the reservoir was filled, in December 1957, the spillway was used to bypass inflow in excess of the capacity of Unit No. 1 while installation of the other two units continued. Discharges up to 8,000 cfs have passed over the spillway. The spillway performance has very closely duplicated that observed in the model tests, and is satisfactory, in all respects. (See photo 9.)

Powerhouse

The outdoor-type powerhouse is a compact structure located in the bed of the river at the toe of the arch dam (Fig. 3). The concrete substructure fills the canyon extending 165 feet from cliff face to cliff face. The three units are spaced 53'-3" on center. To provide an erection and loading area required excavating the basalt cliff on the east bank. The access road to the powerhouse was also bench-excavated in the face of this cliff. The powerhouse was located at a distance downstream of the arch dam which would provide sufficient room between the rear of the station and the dam for the transformer yard. (See photo 7.)

The vertical shaft Francis turbines have a rated capacity of 52,200 hp and an expected full gate capacity of 55,000 hp at 150 feet net effective head; they

PHOTO #9 - GENERAL VIEW - MAR. 1958
Spillway in operation - 3000 cfs

were manufactured by S. Morgan Smith Company. The spiral case is rolled steel-plate construction with field-riveted joints. The runner was made as a weldment of cast and rolled components. Separate unit-type governors were furnished. These cabinet actuators were manufactured by Woodward Governor Company.

The three enclosed, vertical-shaft generators are rated 36,000 kva, 0.90 p.f., 3-phase, 60-cycle, 13.8 kv, 138.5 rpm with amplidyne system of excitation, and were manufactured by General Electric Company. The main guide and thrust bearing is mounted below the rotor; a self-oiling upper guide bearing is also provided. Since the plant is of the fully outdoor type (Fig. 3), the generator enclosure was designed as a weatherproof housing.

A 110-ton gantry crane services the units. A 15-ton auxiliary hoist and a cantilever overhang downstream of the main frame are provided to handle the single set of draft tube gates.

The normal station auxiliary services are all provided comfortably in the relatively small powerhouse.

All electric control and switchgear equipment together with the batteries and machine shop equipment is located in the 13'-6" wide electrical bay on the upstream side of the powerhouse. The control system is semiautomatic to minimize the number of operating personnel required at the plant.

Transformer Yard

The transformer yard is located in the area between the powerhouse and the downstream face of the dam. This area was backfilled with a compacted sand fill to provide a foundation for the equipment. The sand backfill also supports the three, 16-foot diameter penstocks[3] which are buried in the fill under 16 feet of cover.

The three 40,000 kva, single-phase, 60-cycle, forced oil and water-cooled, 13,200-241,500 v main transformers were manufactured by Westinghouse Electric Corporation. Station power is supplied by a 500-kva General Electric Company auxiliary transformer rated 13,200-480 v, 60-cycle, 3-phase, connected to the station bus.

Diversion

During construction of the arch dam and powerhouse, river flow was diverted through a 690-foot-long unlined tunnel excavated through the basalt in the east abutment (Fig. 3). The tunnel had a theoretical design area of 326 square feet but overbreak increased this to approximately 400 square feet. A concrete structure was built to elevation 1,460 at upstream portal. Gate guides for the tunnel closure gate were embedded in the portal concrete. Earth and rockfill cofferdams were extended across the river upstream of the arch dam and downstream of the powerhouse after the diversion tunnel was completed in July 1956. The upstream cofferdam extended some 40 feet above river bed gravel to elevation 1,460 and the downstream cofferdam had a top elevation of 1,440. Water was first diverted in August 1956. During the flood in the last week of February 1957, the tunnel discharged approximately 8,000 cfs before the upstream cofferdam was overtopped as the flow increased to a peak discharge of 10,320 cfs reaching elevation 1479.8 upstream of the partially completed arch dam. The tunnel was closed on November 29, 1957 by lowering the closure gate into place using a mobile crane.

During the period following the closure and prior to the completion of the project, a minimum river flow of 2,500 cfs had to be maintained below the regulating dam at all times to insure adequate flow conditions for the fish in the river. This requirement was met by discharging river flow through the spiral case of the partially erected turbine No. 2 and controlling the discharge with the wicket gates and a temporary oil piping system. The water was bypassed through the No. 2 spiral case and draft tube for approximately one month while the reservoir filled and Unit No. 1 was under test during its trial operation period. Mr. R. A. Sutherland has prepared a paper describing this arrangement entitled, "Free Discharge Through a Hydraulic Turbine Distributor Case and Draft Tube" which will be presented at the annual meeting of the ASME in December 1958.

During the remainder of the construction period, water was passed through Unit No. 1 and over the spillway.

Fish Facilities and Regulating Dam

Some 5.3 million dollars of the nearly 20 million dollar total cost of the Pelton project were spent on the construction of the fish passage facilities and regulating dam. (See photos 4 and 5.)

The regulating dam forms a 2-1/2-mile reservoir with a usable storage capacity of 3,300 acre-feet, and with a high water elevation of 1,435. The

PHOTO #4 - REGULATING DAM & RES. - DEC. 1957
Looking downstream - Reservoir at elev. 1410 -
Fish ladder in right foreground

PHOTO #5 - REGULATING DAM - DEC. 1957
Dam completed - Spillway operating -
Fish ladder in foreground, not yet operating

discharge from the three turbines in the Pelton powerhouse, which will be used as peaking units, will vary from zero cfs for some six or seven hours a day to 11,000 cfs for one to two hours of peak demand. This fluctuating discharge will be stored in the regulating reservoir and then released, as required, to restore the river flow to the natural flow entering the reservoir behind the arch dam.

The 800-foot-long, non-overflow section of the regulating dam is of the rock-fill type with a vertical, impervious central core (Fig. 5). Upstream and downstream filter zones were provided adjacent to the core. The rock-fill section has a maximum height of 80 feet where it abuts the center concrete spillway. All construction materials were obtained by stripping or excavation in the close vicinity of the dam.

The east abutment of the dam above the top of the John Day tuff at approximately elevation 1,400 consisted of strata of coarse gravel, sand and pumice. To increase the path of percolation and reduce seepage velocities to safe limits, the impervious core founded on the tuff, was carried 130 feet into the abutment by excavation. This cut had a maximum height of 100 feet and was backfilled, after placing the core, to elevation 1,445.0 (Fig. 5).

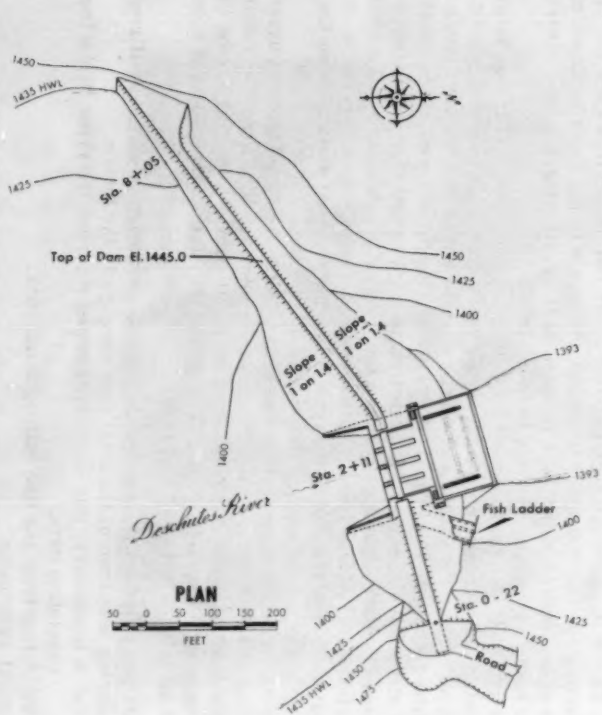
At the narrow point in the river, the west abutment consisted of a huge mass of landslide debris and open talus overlying the John Day tuff. Therefore, the west embankment of the dam was rotated upstream along the flank of this debris and founded on John Day tuff except for the extreme west 180 feet which was built on a stable sand and gravel formation overlying the tuff. The head on this extreme 180 feet was less than 15 feet at full reservoir elevation.

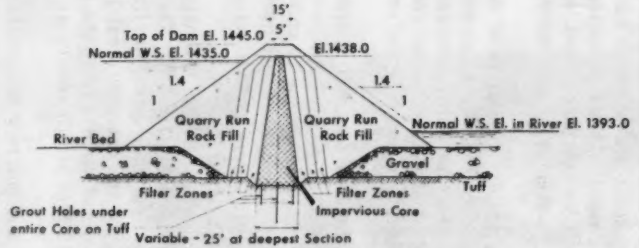
Foundation grouting consisted of consolidation and one row of curtain grout holes, 30 feet deep, in the John Day tuff under the impervious core and spillway section. Some leakage through the abutments to the river is expected; this is acceptable in view of the purpose of the regulating dam and reservoir and no loss in power or revenue is involved.

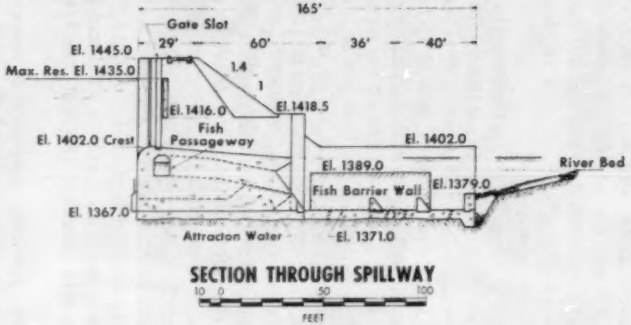
The concrete spillway section has a crest elevation of 1,402; discharges are controlled through four 20-foot-wide by 14-foot-high, wheeled, vertical-lift gates. Above the gates, a concrete curtain wall spans between piers from elevation 1,416 to 1,435 (Fig. 5) to reduce the height and cost of the gates, hoists and hoist structures. This design was developed since, to obtain sufficient usable reservoir capacity, the usable range of storage had to extend from elevation 1,408 to 1,435. With this unusual spillway arrangement, all spillway releases are a sluicing type of discharge, even when passing flood flows up to the maximum design value of 30,000 cfs.

The shape of the ogee section together with the fish passageways which are embodied in the structure further add to its uniqueness. These fish facility features have been described in other publications,[4,5] which describe the fish passage facilities in detail. Therefore, only the following list of component structures will be given here:

1. Initial Provisions - for use during the construction period or if required during shutdown of the final passageways in the future.
   (a) A temporary, pile-supported barrier across the river below the regulating dam.
   (b) A fish trap and fish hauling tank truck.
2. Final Provisions
   (a) Entrance and/or exit structures incorporated in the regulating dam concrete spillway section.







GENERAL PLAN AND SECTIONS REGULATING DAM

FIGURE 5

(b) 2.78 miles of fish ladder rising 184 feet.
(c) A 300-foot long drop structure section of the ladder just below its junction with the arch dam to accommodate fluctuations in the main reservoir level up to a maximum of seven feet.
(d) A "skimmer" and pump house structure upstream of the arch dam to recirculate water for attacting and collecting downstream migrant fingerlings, and a connecting pipeline to pass these fingerlings into the ladder downstream of the drop structure.

The fish ladder is the longest ever constructed and has one of the highest lifts known. The "skimmer" structure is the first structure of this type to be built, although this method of attracting and collecting downstream migrants has been under study for some time.[2,5]

Construction Plant

The Pelton project required some 94,800 cubic yards of concrete which was placed as follows:

Diversion tunnel	=	1,400 cubic yards
Powerhouse and tailrace	=	14,200 " "
Arch dam	=	39,000 " "
Spillway, Thrust block, Gravity sects. and Intake cantilevers	=	7,100 " "
Regulating dam spillway	=	23,400 " "
Fish facilities	=	9,700 " "

All told, some 300,000 cubic yards of rock and unclassified material had to be excavated.

To meet these requirements, a complete construction plant was established at the job site. The basic elements were:

1) A 2,000-foot cableway with a movable head tower
2) Quarry and aggregate plant
3) Concrete batching and mixing plant
4) A 3,500-cfm compressor station
5) A chlorinated water supply system
6) A shop, storage, and form panel area together with office space.

No living facilities were provided at the job site for either workers or supervisory personnel.

Cableway

To transport and place construction equipment, forms, concrete and other items in the vicinity of the arch dam and powerhouse, a cableway was installed in 1956 (Fig. 2). The movable head tower was erected on the east bank, mounted on rails. The track at elevation 1,860 was 600 feet in length and formed a flat arc in plan. A fixed concrete anchor block was constructed on the west bank just below the rim of the canyon at approximately elevation 2,000. A 3-1/4-inch diameter main cable extended approximately 2,000 feet

across the canyon. Nearly 200 cubic yards of concrete were placed as the head tower counterweight. The Lidgerwood hoist installed in the head tower had a 35-ton capacity. One of the more spectacular uses of the high line occurred shortly after its completion when it transported a 27-ton Caterpillar D-8 tractor with blade and ripper across the canyon.

The hoist was remotely controlled from a booth near the concrete batch plant (elevation 1,600). The operator at this control center was in communication by hand signal, bell, telephone and radio telephone with men stationed on the dam or in the powerhouse; in addition, a rigger rode a platform just below the block and was responsible for securing all items to be hoisted.

The cableway, as installed, could reach every part of the arch dam and powerhouse construction area except a small area at the downstream end of the spillway chute.

Quarry and Aggregate Plant

A quarry was uncovered about 1/4 mile off the main access road, approximately 1-1/2 miles from the arch dam. The material in the quarry site was classified as a hard, dense basalt which is in the transition phase between basalt and andesite. The rock was quarried by drilling and blasting, and loaded by a 1-1/2-yard shovel into 12-yard rear dump trucks which transported the rock to the aggregate processing plant.

The aggregate processing plant was located alongside the main access road 1-1/4 miles from the arch dam. A total of 170,000 cubic yards of aggregate was required for the 94,800 cubic yards of concrete, and five sizes were processed from the basalt: 6 to 3 inch, 3 to 1-1/2 inch, 1-1/2 to 3/4 inch, 3/4 inch to No. 4, and sand.

The flow of rock from quarry to finished aggregate was as follows:

1. Quarry rock was dumped into a primary jaw crusher.
2. The crushed rock was conveyed to the first set of screens to scalp dirt and fines,
3. Passed through a gyratory crusher,
4. Conveyed to the second set of screens which separated the 6", 3" and 1-1/2" maximum sizes into bunkers.
5. A portion of the 1-1/2" material was carried by conveyor to a flat head crusher which produced 3/4" material.
6. The 3/4" material was dumped back on the conveyor, went through the screens and was deposited in a surge pile over a reclaiming tunnel;
7. The 3/4" rock was delivered through the reclaiming tunnel to screens which separated the 3/4" to No. 4 size from the finer material and deposited it into a bunker.
8. A portion of this finer material passed through a roll mill, a rod mill and finally a classifier for the manufacture of sand, which was stockpiled.

Water was supplied from the river by pumps through an 8-inch pipeline and was discharged into a small creek near the quarry behind appropriate check dams which clarified the discharge water before re-entry into the Deschutes River.

The output of plant varied between 1,000 and 1,600 cubic yards of aggregate per day with the rod mill running 24 hours and the remainder of the plant operating on an eight-hour shift.

Production at the processing plant started in September 1956. Material was trucked from the plant to the aggregate storage area located on a bench excavated on the east bank above the concrete batch plant (Fig. 2). The trucked aggregate was dumped into a concrete reclaiming hopper near the cableway head tower. From this point it was transported by conveyor to a shuttle conveyor and distributed by size into one of the five stockpiles over a reclaiming tunnel on a benched area downhill. Sufficient aggregate for approximately 2,000 cubic yards of concrete was maintained in the area. Gates in the reclaiming tunnel beneath the five stockpiles were of the air ram type controlled from the batch plant. The tunnel conveyor carried the coarse aggregate to a double deck shaking and rinsing screen where the minus No. 4 material was scalped and wasted. The fine aggregate bypassed this screen feeding directly to the batch plant charging conveyor.

During the early period of aggregate handling, control sampling indicated an excessive percentage of undersize material in all sizes of coarse aggregate. This condition was corrected primarily by adding rock ladders below the shuttle conveyor feeding the aggregate stockpiles and by adjustment at the processing plant.

Concrete Batching and Mixing Plant

Alongside the aggregate stockpiles a 7,500-barrel cement silo was erected to receive the Type II cement trucked in from Oswego, Oregon. The silo was charged by an air slick, and the cement was fed to the batch plant located at approximately elevation 1,600 just upstream of the arch dam and downhill from the silo and stockpiles (Fig. 2).

The batch plant, manufactured by the Noble Company, was a fully automatic type Model CA-355. The single, 350-ton, five-compartment aggregate bin was equipped with three dribble-type and two cobble gates, all operated by automatically controlled air rams. The cement silo had a capacity of 2,000 cubic feet. The automatic batching system included individual weigh batchers for sand, 3/4-inch, 1-1/2-inch, 3-inch and 6-inch rock, cement and water and an automatically controlled air-entraining agent dispenser interlocked with the water scale. The mix selector was capable of selecting any of six predetermined mixes. Indicating lights enabled the operators and inspectors to observe the status of the batching, and automatic recording devices provided a continuous record. The two 2-yard tilting mixers installed in the plant were charged by an automatically controlled charging chute and appurtenances. The wet-mix holding hopper had a nine-yard capacity with an air operated discharge gate. The drive-through clearance under this holding hopper was 16 feet.

For placement by high line the mix was deposited into four-yard buckets on flatcars. The flatcars were moved on a shuttle track by dinkey engine from under the batch plant to a point in line with the concrete placement area to minimize traveling the cableway head tower.

For the regulating dam concrete, the wet mix was transported by dumpcrete trucks which deposited the mix into two-yard buckets which were placed by a gantry crane erected inside the cofferdam. This crane had a 9-ton lifting capacity with a boom radius of 125 feet; a jib on the boom extended this radius to 140 feet.

Concrete for the fish ladder was transported from the batch plant in agitator trucks. The mix was deposited from buckets placed by a mobile crane.

Concrete Control

A concrete laboratory was set up near the batch plant to control the concrete quality. The quality control program was intensive and covered the entire span of the construction. This control began with tests of samples from probable aggregate sources and design of trial mixes. It continued through all phases of the work, with the laboratory reports covering,

1. The mechanical and physical tests of concrete aggregates and ingredients.
2. Physical tests of plastic concrete.
3. Physical tests of hardened concrete.

Inspection covered the mixing, transporting, placing, curing and finishing of the concrete. In all, some 1,300 concrete test cylinders were cast and broken, covering 97 variations of the four classes of concrete which were specified: Class A, 4,000 psi at 91 days; Class B, C and D, 3,000 psi, 2,500 psi and 2,000 psi at 28 days respectively. All concrete was air-entrained.

Temperature control of the concrete presented no problem. The adoption in the design of the relatively thin section for the arch dam resulted in most massive pour on the job being only 530 cubic yards. Nevertheless, close inspection of placing temperatures was maintained and embedded thermometers were used to record temperatures after placing.

Trial Operation

Following closure of the diversion tunnel on November 29, 1957, Unit No. 1 was completed and ready for trial operation on December 11. During this period, the station service systems were checked out and put in operation. The start-up was under the supervision of an experienced hydro-plant design engineer who acted as over-all coordinator. The manufacturer's representatives and responsible construction personnel acted under the coordinator's supervision. On December 11, the mechanical trial run of the turbine and generator began and in sequence the generator dry-out, high-potential test and phasing out of the generator followed, and were completed on December 19. On the same day load rejection tests were run and satisfactorily completed. The unit was synchronized and connected to the line for commercial operation on December 20, 1957, followed by Unit No. 2 on March 12, 1958 and No. 3 on April 14, 1958.

ACKNOWLEDGMENTS

The Pelton project was designed and the construction was supervised by Ebasco Services Incorporated, representing the Portland General Electric Company. The design and the supervision of construction of the transmission lines was performed by Portland General Electric Company. The hydraulic spillway model was constructed and tested at the Alden Hydraulic Laboratory of the Worcester Polytechnic Institute. Guy F. Atkinson Company was the prime contractor. The Oregon Fish and Game Commissions, the United States Fish and Wildlife Service and Mr. Harlan B. Holmes all rendered valuable assistance in the design of the fish facilities.

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2. "Coexistence of Fish and Dams," H. A. Preston & L. E. Rydell, ASCE Proceeding Paper No. 1414, October 1957.

3. Latham, G. R., "Penstock Design and Construction," ASCE Proceedings Paper No. 1285, Page 19.

4. Pearce, C. E., "The Pelton Fish Ladder" a paper presented before the NELA Hydraulic Committee May 12, 1958.

5. Eicher, G. J., "PGE Pioneers New Approach to Fish Passage at Pelton Project" Electric Light and Power March 15, 1958, pages 56-60.

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POWER DIVISION
Proceedings of the American Society of Civil Engineers

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Penstock Experience and Design Practice, by Gordon V. Richards. (Proc. Paper 1397, October, 1957. Prior discussion: 1618. Discussion closed.)
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Large Spiral Casings of T-1 Steel, by E. L. Seeland. (Proc. Paper 1398, October, 1957. Prior discussion: 1618. Discussion closed.)
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Fixed-Wheel Gates for Penstock Intakes, by Sylvan J. Skinner. (Proc. Paper 1420, October, 1957. Prior discussion: 1618. Discussion closed.)
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Circulating Water Systems of Steam Power Plants, by R. T. Richards. (Proc. Paper 1488, December, 1957. Prior discussions: 1618, 1689. Discussion closed.)
by R. T. Richards (closure) 1830-17

Ambuklao Underground Power Station, by Andrew Eberhardt. (Proc. Paper 1598, April, 1958. Prior discussion: none. Discussion closed.)
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Note: Paper 1830 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, PO 5, October, 1958.

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Closure by J. Barry Cooke and Arthur G. Strassburger

J. BARRY COOKE,[1] M. ASCE and ARTHUR G. STRASSBURGER,[2] J. M. ASCE.—The authors wish to express their appreciation of the comments, additions, and corrections submitted by Messrs. Ospina, Whetstone, and Nicolaou.

It has been suggested that the Section C listing of basic data might have been enhanced by giving excavation dimensions for the housing of valves, transformers, etc. These and other comments make it appear worthwhile to express the authors' limited purpose in providing the List of Plants. The List of Plants is cross-referenced with the References to assist the reader in locating references for a specific plant. The physical data is to help the reader locate a plant that may be similar to one under consideration. The plants listed with no references are included in an attempt to provide as comprehensive a listing of the world's plants as possible.

The translation of reference number 131, page 17, should read "The General Problem and the Method of Construction of the Underground Power Stations".

The authors wish to thank those who have advised them of corrections and additions to the listing of plants, Section C. In particular, the complete listing of plants furnished by Dr. Ing. Carlo Semenza on the plants in Italy is valued. The following revisions to Section C are in two separate lists: (1) corrections, and (2) additions—

a. Proc. Paper 1350, August, 1957, by J. Barry Cooke and Arthur G. Strassburger.

1. Supervising Civ. Engr., Pacific Gas and Electric Co., San Francisco, Calif.
2. Engr., Pacific Gas and Electric Co., San Francisco, Calif.

Country and Plant (1)	Turbines No. and Type (2)	Plant Capacity MW (3)	Static Head Ft. (4)	Powerhouse Chamber L x W x H (5)	Reference Number (Section D) (6)	Year Initial Oper. (7)
MEXICO						
Tingambato	3 Fv	150	1127	131 x 55 x 114		1957
RHODESIA						
Kariba (right)	6 Fv	600	300	460 x 90 x 125	199	1960
Kariba (left)		600	300		199	Future
SCOTLAND						
Ceannacroc	2 Fv	20	299	92 x 45 x 82 (48)	85, 173, 180	1955
Clachan (cut and cover)	1 Fv	40	965	80 x 56 x 75(53)	45, 173, 180, 187, 192, 213	

The following tabulations are added to Section C.
The complete tabulation now includes 296 plants.

AUSTRALIA						
Great Lakes (Tasmania)	6 Pv	300	2750	300 x 55 x 100		196-
AUSTRIA						
Braz	3 Ph	30	1000	210 x 55 x 65		1953
Prutz-Imst	3 Fv	87	475	184 x 66 x 85(57)		1957
CHINA (FORMOSA)						
Ku-Kuan	4 Fv	180	500			1961
ITALY						
Acri	2 Ph	110	2090	256 x 87 x 82(44)		1953
Arsie	2 Fv	48	463	108 x 56 x 111(43)		1955
Bresa	1 Ph	4	371	59 x 25 x 30		1941
Bressanone	5 Fv	142.5	538	272 x 67 x 108(55)		1940
Brunico	2 Fv	21	672	96 x 63 x 104(42)		1958
Cavilla	2 Fv	35	269	111 x 41 x 105(43)		1954
Chiavenna	3 Fv	60.5	1100	115 x 46 x 89(46)		1949
Cividate (pit type)	3 Fv	52.5	428	205 x 66 x 79(24)		1945

Colomber	1 Fh	10	438	85 x 61 x 73(43)		1959
Commacqua	2 Fv	10	444	74 x 30 x 71(34)		1942
Flumendosa #1	1 Fh	8	132	85 x 25 x 35	7, 51, 131	1949
Flumendosa #3	2 Fh	10	561	97 x 30 x 40(35)	7, 51, 131	1949
Gerola Alta (buried)	1 Fh	13.5	2350	64 x 50 x 66 (32)		1942
Grigno	2 Fh	12	1665	127 x 40 x 47(29)		1942
Grosio	4 Fh	400	1962	400 x 68 x 111(49)		1959
Isolato	2 Fh	43	2145	138 x 43 x 71(36)		1953
Malgo Boazzo	2	96	1838			196-
Medumo (pit type)	3 Fv	14	234			1952
Montelungo (1/2 under)	2 Fv	32.5	460	94 x 59 x 87(43)		1953
Morasco	2 Fh	45	2100	129 x 54 x 68(33)		1953
Nembia	1 Kv	13.5	289	x 206(135)		1957
Pian Sule'	1 Fh	7	541	69 x 34 x 39		1931
Pantano D'Avio	2 Fh	14	1465			1955
Pontesei	1 Fh	6	122	97 x 38 x 36		
Regoledo	2 Fh	36	1583	147 x 39 x 115(33)		1941
Salarno	3 Fh	4.5	162	118 x 16 x 21		1940
Salisano	2 Fh	16	805	157 x 49 x 61(35)		1940
San Floriano D'Egna	3 Fh	195	1875	256 x 91 x 84(42)		1956
San Silvestro	1 Fv	60	1477	98 x 45 x 89 (48)		1958
Scalelle	2 Fv	8	87	71 x 29 x 40		1942
Signayes	3 Fh	43	1140	164 x 56 x 56(39)		1951
Triano (1/2 under)	3 Fv	26	158	250 x 89 x 125		1942
Villa Santa Maria	3 Fh	73	1392	322 x 64 x 74(40)		1952
Vinadio (pit type)	2 Fv	61	1263			1956
Vizza (cutaway)	1 Kv	5.5	45	131 x 28 x 41		1940
FINLAND						
Pirttikoski	2 Kv	110	89	194 x 62 x 50		1959

The following revised tabulations appear in Section C

Country and Plant (1)	Turbines No. and Type (2)	Plant Capacity MW (3)	Static Head Ft. (4)	Powerhouse Chamber L x W x H (5)	Reference Number (Section D) (6)	Year Initial Oper. (7)
ALGERIA						
Darguinah (Ahrzerouftis)	1 Pv	5	555	239 x 106 x 85	154, 173, 176, 180	1951
AUSTRALIA						
Barren Falls	3 Fh	3.8	420	84 x 27 (23)	25, 27, 32, 34, 96	1935
Kieva No. 1	6 Pv	96	1730	244 x 45 x 78	28, 29, 30	1961
Kieva No. 4	4 Fv	62	653	130 x 40 x 72	28, 29, 30, 183	1955
M-6 (Snowy Mt.)		920±		112, 210		196-
M-7 (Snowy Mt.)		200±		112, 210		196-
T-1 (Snowy Mt.)	4 Fv	320	1100	305 x 59 x 105	32, 101, 112, 132, 147, 173, 180, 189, 210	1959
T-2 (Snowy Mt.)	4	280	940	320 x 51 x 107	112, 210	1962
Tully Falls No. 2	4 Fh	72	1485	230 x 50 x 66	183	1957
BRAZIL						
Cubatao	4 Pv	260	2230	392 x 67 x 127	122, 156, 165	1956
	2 Pv	130	2230		171, 180, 194	196-
Paulo Afonso	3 Fv	180	265	192 x 48 x 102		1954
	6 Fv	360	265	360 x 59 x 118		Ult.
EGYPT						
High Aswan	16 Fv	2880	305	2 chambers, ea. 920 x 72 x 100 (80)		196-
FRANCE						
Brommat	6 Fv	195	853	262 x 72 x 103	9, 51, 57, 121, 173, 180	1932
Peyrat-le-Chateau	3 Fv	55	830	138 x 46 x 57	64	1952
Tolla	2 Fv	9.3	682	134 x 36 x 36		196-
Valabres	2 Fv	30	664	118 x 36 x 63		1957
INDIA						
Konar			E l i m i n a t e			
ITALY						
Agordo	2 Fv	26	492	114 x 50 x 92(53)	35, 41, 96	1940

Ampezzo (Lumiei)	3 Ph	60	1578	236 x 51 x 83(44)	48, 131, 173, 180	1948
Ala	2 Kv	45	92	145 x 44 x 132(54)	75, 157	1953
Avise	3 Ph	147	3400	250 x 85 x 87(56)		1954
Barcis (cutaway)	2 Kv	20	201	89 x 47 x 118(35)		1954
Belviso Inferiore	2 Ph	70	1796	304 x 54 x 84(45)		1947
Castelbello	3 Fv	85	968	133 x 59 x 108(59)	173, 180	1949
Cencenighe	2 Fv	30	663	77 x 38 x 96(54)	51, 131	1939
Coghinas	4 Fh	27	322	269 x 49 x 72(39)	(1st in Italy) 51, 131	1927
Cotilia (buried)	2 Fh	60	479	305 x 118 x 92(59)		1942
	2 Kv	6	105			
Flumendosa #2	3 Ph	30	1370	207 x 49 x 51(45)	7, 51, 131	1949
Gardona	1 Fh	15	493	115 x 48 x 67(36)		1955
Glorenza	2 Ph	100	1930	187 x 65 x 80(37)		1948
Hone (cutaway)	1 Kv	18.5	121	75 x 43 x 130(62)		1947
Lana	3 Ph	120	1593	220 x 56 x 82(57)		1953
Lovero (cutaway)	2 Fv	44	361	176 x 62 x 102(49)		1948
Malgo Ciapela	1 Ph	20	1860	105 x 47 x 71(40)		1956
	3 Pumps				63, 173, 180	
Mera	3 Fv	73	1080			
Monte Argento	3 Fv	66	169	210 x 43 x 111(49)	51, 67, 173, 180	1950
Montorio	3 Fv	116	863	207 x 74 x 107(62)	51, 67, 173, 180	1955
Narni	2 Kv	45	92	193 x 69 x 114(44)		1958
Ponte Gardena	3 Fv	55	200	163 x 62 x 92 (38)		1938
Pontey	2 Fv	34	228	119 x 67 x 121(59)		1950
Premadio	5 Ph	375	2320	483 x 98 x 100(59)		1955
Provvidenza	3 Fh	150	878	392 x 62 x 82(51)	51, 173, 180	1949
Quart	2 Fh	40	247	154 x 63 x 76(57)		1958
San Giacomo	3 Ph	165	2175	329 x 53 x 79(48)	51, 173, 180	1947
Santa Guistina	3 Fv	104	606	225 x 70 x 111(55)	63, 84, 173, 180	1951
Santa Massenza	6 Ph	355	1935	632 x 95 x 92(52)	63, 121, 173, 180	1953
Somplago	5 Fv	275	950	296 x 82 x 120(57)		1957
Sondrio	2 Fv	140	2090	143 x 85 x 97(46)		1959
Sonico-Cedegolo	3 Fv	74	769	154 x 55 x 92(45)	63, 173, 180	1950
Soverzene	4 Fv	220	930	243 x 83 x 129(60)	78, 173, 180	1951
Stanga	2 Fv	30	500	189 x 62 x 101(52)		1943
Stazzona	2 Fv	35	355	111 x 56 x 102(48)		1938
Zappello	2 Fh	10	1312	153 x 31 x 47(29)		1931
JAPAN						
Sudagai	2 Fv	46	274	116 x 55 x 102(55)		1955

PENSTOCK EXPERIENCE AND DESIGN PRACTICE[a]

Closure by Gordon V. Richards

GORDON V. RICHARDS,[1] M. ASCE.—Mr. Lawton's observation regarding the Company's first substantial use of welded field joints since World War II is correct. It should be pointed out, however, that conditions at Haas are extraordinary and the conclusion that the Company has abandoned rivets in favor of welding on all penstocks should not be made.

With regard to the possibility of brittle fracture, the Company's practice conforms to the ASME Boiler Code for Unfired Pressure Vessels which does not require impact testing for service above -20° F or for service below that temperature when due to lower seasonal atmospheric temperature. From available information,* it would appear that the transition temperature of A-212 steel after welding is a rather uncertain figure, but could well be higher than the ordinary temperatures of penstocks. The fact that no failures have occurred, brittle or otherwise, to the Company's penstocks suggests that the penstocks are actually safer than is indicated by tests of notch-bar resistance.

Damage to penstocks in handling has never been a major problem with the Company. There have been instances where sections of pipe have been bumped enough to loosen pieces of the coal-tar lining. This has been replaced in the field by hand daubing. On other occasions, ends of pipe sections have been marred sufficiently to interfere with entry of the riveted joint, but correction has always been possible with a minor amount of grinding. In the event of major damage to a penstock section, it would be necessary to return the section to the shop for correction. Fortunately, this has never been found necessary.

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a. Proc. Paper 1397, October, 1957, by Gordon V. Richards.

1. Sr. Civ. Engr., Pacific Gas and Electric Co., San Francisco, Calif.

*For example, "Transition Temperature Behavior of Pressure Vessel Steels", by L. J. McGeady, The Welding Journal, January, 1955.

LARGE SPIRAL CASINGS OF T-1 STEEL[a]

Closure by E. L. Seeland

E. L. SEELAND,[1] M. ASCE.—Mr. Lawton's discussion is deeply appreciated. Welding costs have not been substantiated as yet by field experience. The appraisal of the excellent property of notched toughness was based on both the T-1 steel behavior during welding operations and service conditions.

This is our initial installation using T-1 steel and the spiral casings are still to be constructed. Some saving is estimated in the use of T-1 steel over mild steel. However, the amount of savings is, as yet, undetermined. No breakdown is available regarding the shipping, material and welding savings.

The use of T-1 steel in spiral casings resulted in a reduction in weight of approximately 37%. It is not possible to make the anticipated 50% saving as the minimum thickness of 1/2 inch was established for field handling.

Reduced section thickness is an important consideration in the selection of T-1 steel, particularly since it reduces the amount of field welding. Welding of T-1 steel of the same thickness will cost more than welding of mild steel. However, a savings may be effected in the welding of the equivalent thinner T-1 steel plates.

During the past 10 years substantial advances have been made in the understanding of low-energy or brittle failures.

T-1 steel's excellent property of notched toughness at low temperatures is of considerable importance in the selection of this steel for use in spiral casings. Assurance must be obtained that the material will not break in a brittle, glass-like fashion or burst in a catastrophic manner within or above the operating temperatures.

The United States Steel Corporation and the Chicago Bridge and Iron Company jointly conducted a series of tests on T-1 steel for pressure vessel construction. These tests demonstrated two main advantages of T-1 steel, as follows:

1. Stress relieving is not necessary on T-1 steel even though welding is involved.
2. The vessels tested at -50° F withstood hydrostatic pressures of 130,000 psi before bursting and when rupture finally occurred, it did so in a completely ductile manner.

The United States Steel Corporation made further tests on 1/2-inch-thick T-1 steel plate welded with E-12015 electrodes to determine the ductility in transition temperatures. The transition temperature used was the temperature at which 50% of the fractures occurred in the shear mode, i.e., 50% of

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a. Proc. Paper 1398, October, 1957, by E. L. Seeland.

1. Superintendent of Design, Ebasco Services, Inc., New York, N. Y.

the fracture is fibrous. On this basis, the transition temperature of -42° F was established as the point where ductile fracture is converted to brittle fracture.

As the operating temperatures for the spiral casings are approximately + 35° F during operation and + 70° F when empty, we feel that we have taken reasonable precautions in the selection of a material that would not fracture in a glass-like fashion at any possible operating temperature.

Heavy low-carbon steel plates above 1 inch thick are difficult to weld in the field, and the many multiple layers of welding increase the chance of porosity, slag inclusion and lack of penetration. In plates nearing 2 inches thick, special provisions must be made for a preheat, continuous heat during welding, and post-heat treatment. Owing to the size of these casings, all of these special heat treatments must be performed in the field where it is most difficult to control any type of heat treatment of the steel.

We do not feel that the selection of T-1 steel is a cure-all for all of our past problems, but it is a step in the direction of providing economical, safe spiral casings for hydro stations with increasing heads and capacities.

COEXISTENCE OF FISH AND DAMS[a]

Closure by Howard A. Preston and Louis E. Rydell

HOWARD A. PRESTON,[1] M. ASCE and LOUIS E. RYDELL,[2] M. ASCE.—In preparing this paper the authors attempted to present a broad picture of a problem that is vital to the development of the water resources of the Pacific Northwest. The problems encountered, and the complexities of the engineering solutions are such as to create a new and specialized field for civil engineers; a field where engineers must work in close partnership with fishery biologists, and where research assumes major importance. Also presented is a summarized progress report on accomplishments up to mid-1957 in achieving solution of various specific phases of the problem.

The constructive discussion of the paper by Mr. Benson is appreciated. His suggestion that it would have been desirable if the authors could have presented a solution for each case is certainly concurred in, but as he states, this still lies in the future. His suggestion regarding the possibility of improving safe passage of fingerlings through turbines by design modifications to reduce the negative pressures has considerable merit. This can be accomplished by lowering of the turbine setting which will in many cases also result in improved power characteristics, but of course, at increased costs. This factor has been a consideration in the design of the Kaplan type turbines for Ice Harbor Dam on Snake River, where the distributor centerline is placed approximately 16.5 feet below normal minimum tailwater. The increased cost was justified by power benefits alone, with the added benefits to fish passage thrown in for good measure. McNary Dam turbines, now about 8.5 feet below tailwater, will eventually be about 22.5 feet below the normal minimum pool of John Day Dam, the next downstream project now under construction. Other points developed by Mr. Benson are well taken.

Although informal comments on this paper have been received from some of our partners in this work, the fishery biologists, it is to be regretted that formal discussions from their point of view have not been presented for publication.

Since preparation of this paper, two high-head power developments involving novel fish passage facilities have been substantially completed in the Pacific Northwest. Pelton Dam on Deschutes River in Oregon, with an effective height of about 188 feet and utilizing the "Skimmer" principle for collection of downstream migrants, has recently been placed in operation.

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a. Proc. Paper 1414, October, 1957, by Howard A. Preston and Louis E. Rydell.

1. Chf., Flood Control Reports Branch, Corps of Engrs., U. S. Dept. of the Army, Walla Walla, Wash.
2. Chf. Planning & Reports Branch, Corps of Engrs., U. S. Dept. of the Army, Walla Walla, Wash.

Brownlee Dam in the Snake River Canyon between Oregon and Idaho, with an effective height of about 280 feet and utilizing a huge net with 1/8 inch mesh extending across the reservoir to a depth of 120 feet for interception of fingerlings, is nearing completion. Active research, with some very encouraging preliminary results, is also under way to improve the design of the collector units for downstream migrants, and on methods of separating the fingerlings from the large quantities of attraction water being used. These studies, initiated at Mud Mountain Dam on White River, Washington, have been transferred to Baker Dam on Baker River, Washington, where the results of this study will be used in another high-head hydroelectric project under construction upstream therefrom. Preliminary findings on some basic research in fish behavior indicate that the element of fatigue for adult salmon ascending ladders is not as critical as previously judged. These and other findings are permitting major revisions in the size and slope of fish ladders being currently designed for the Lower Snake River Dams.

It is evident that considerable progress, in research and as well as in actual construction of new types of facilities, has been made since preparation of the paper and is continuing.

FIXED-WHEEL GATES FOR PENSTOCK INTAKES[a]

Closure by Sylvan J. Skinner

SYLVAN J. SKINNER.[1]—The writer appreciates the views presented by Mr. Joseph R. Bowman and the points he raises are worthy of further consideration. Some of the points discussed, however, appear to need some clarification.

The additional data on the comparative merits of Coaster (Caterpillar) v.s. Fixed-Wheel Gates are well presented by Mr. Bowman. The problem of eccentric loading of rollers due to deflection and end rotation of the gate leaf is evident, however, a successful and fairly simple method of correcting this condition consists of supporting the roller tracks (those on the gate leaf and on the embedded track base) on confined hard rubber mats. The mats act as hydraulic supports and distribute the roller loads laterally and vertically, thus eliminating the high roller end load condition cited. This construction will also tend to remove the need for restricting the gate leaf width, which may have been considered necessary with high end-loaded rollers.

It is possible that failure of some Coaster (Caterpillar) gates to operate properly results from assuming too low a coefficient of friction, as stated in the discussion; but a major portion of the trouble is probably caused by an excessive accumulation of debris and deposits on the links and rollers after having been submerged for an extended period of time. Frequent inspection and servicing of such installations is needed.

Estimating Data

Mr. Bowman's comments regarding parameter requirements of weight curves for estimating purposes are correct when considering all types of gates and full range of operating heads. The writer, however, in the basic paper discusses penstock intake high head fixed-wheel gates for which the parameters of Fig. IV are applicable. For other types of installations or installations at lower heads the writer uses other applicable weight curve charts. The magnitude of covering all installation variations would extend the scope beyond the intent of the present paper.

Design Details

The writers fixed-wheel gate designs incorporate completely welded, riveted or bolted structures, or combinations of these methods of construction. Completely welded gates requiring very heavy weld deposits for large, maximum stressed gate members are avoided. Distortion due to shop welding,

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a. Proc. Paper 1420, October, 1957, by Sylvan J. Skinner.

1. Mech. Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

even when using the submerged arc welding process with twin electrodes (considered the least objectionable from a distortion standpoint), is difficult to control to the degree necessary for maintaining the required accuracy for alinement of members. Stress relieving is a definite requirement for such structures and can produce distortion or aggravate that which already exists from the welding process. The large, heavy, multiple unit gates are therefore usually specified to be fabricated using high strength ribbed bolts with self-locking nuts. This method has proven to be quite satisfactory.

Mr. Bowman is due an expression of thanks for his constructive discussion of the paper.

Errata "Paper 1420"

Page 1420-8 line 16 from top of page, delete the word 'applied' and insert 'applies'.

Page 1420-10, line 9 from top of page, delete last word 'of' and insert 'or'.

Page 1420-15, under 'UNIT STRESSES', line 7 delete last word 'were' and insert 'where'.

Page 1420-30, first line delete formula '$M_t = P/r\lambda$ track' and insert '$M_t = P/4\lambda$ track'.

CIRCULATING WATER SYSTEMS OF STEAM POWER PLANTS[a]

Closure by R. T. Richards

R. T. RICHARDS,[1] A. M. ASCE.—Mr. Bolieau has given an interesting and informative review of many of the hydraulic details used in the circulating water systems of an important group of steam electric stations. Such general information concerning current practice is always of value to the designer.

Mr. Barr's discussion points out that there is much more to the design of steam plant circulating water systems than the several hydraulic features set forth by the author. This is indeed true and must be considered in the complete design. However, the author is concerned here particularly with the work of the hydraulic engineer who is responsible for moving a particular quantity of water to and from locations selected in studies by others. Neglect of this one phase of pumping plant design can and often does undo some of the fine work contributed to the project by other civil and mechanical engineers.

For an elaboration of some of the problems of circulating water system air binding briefly mentioned in this paper, the reader's attention is invited to ASCE Proceedings Paper 1454, December 1957.

a. Proc. Paper 1488, December, 1957, by R. T. Richards.
1. Civ. Engr. Ebasco Services Inc., New York, N. Y.

AMBUKLAO UNDERGROUND POWER STATION[a]

Discussions by F. C. Rodriguez and Stavros N. Nicolaou

F. C. RODRIGUEZ.[1]—The author has discussed several interesting features of this project. The most unusual is the adoption of horizontal units arranged radially. This is probably the first underground power station where such a setting of the turbines has been adopted. There are other projects wherein a radial arrangement has been used such as at Chastang Project in France but these are above-ground power stations and the units are vertical.

The author believes that the horizontal shaft setting of the turbines appears to be particularly advantageous for underground power stations. The most significant advantage is the savings in excavations and concrete. The writer cautions, however, that a definite choice between a horizontal and vertical setting for any particular project should be made only after an exhaustive comparison of the cost. This cost comparison must take into consideration the following factors among others:

1) The volume and total cost of excavation in the main chamber for each setting. There is no doubt that the horizontal setting will require a lower height of the chamber. It is doubtful, however, if the width of the chamber will be materially different for each setting and it is quite probable that the length for a vertical setting will be less than that for a horizontal setting.
2) The volume and total cost of the excavation in the turbine vaults and draft tubes. In determining the cost due consideration must be given to the difficulty in the excavation in these particular areas due to the relatively slender columns of rock left in place as shown in Fig. 10 (Vol. 84, No. PO 2, Journal of the Power Division.)
3) The possible savings or increase in cost of the penstock liners.
4) The volume and total cost of the concrete.

The other unusual features of this project are the following:

1) An underground valve chamber separate from the power chamber.
2) The utilization of a section of the diversion tunnel as part of the power conduit.
3) The adoption of a "submarine" tower intake. One important consideration in the choice of this type of intake is the absence of any forest or any possible source of large quantities of debris upstream of the dam thereby reducing the necessity of frequent rakings of the trash racks.

a. Proc. Paper 1598, April, 1958, by Andrew Eberhardt.

1. Pres., Engineering & Development Corp. of the Philippines, Manila, Philippines.

4) The complex geology of the site which as the author has pointed out influenced the location of the power facilities structures especially the power station. The power station was located at an area where it was believed the geology of the rock is relatively good. The power station was originally designed without any concrete lining of the walls except at the turbine vaults. The poor quality of the rock exposed during excavation, however, necessitated supporting the roof arch on concrete columns and lining the entire wall as shown in Figs. 9 and 10. This condition of the rock was manifested by a major cave-in in the power chamber.

The author mentioned a major cave-in in the power station roof. This more than any incident during construction attested to the complex geology of the site. A weak zone 10 to 20 meters wide running from the surface down to and beyond the level of the power station intersected a fault about 5 meters above the roof of the power station. The structural weakness of the rock and the temporary build-up of hydrostatic pressure precipitated the cave-in. It is quite significant that on each side of this cave-in, the rock was relatively stable.

This was, however, not the only cave-in that occurred in Ambuklao during the underground excavation work. There was also a minor cave-in in the manifold branch of Unit No. 1 and another major cave-in during the early part of the excavation in the access tunnel. These occurred at sections where the same weak zone intersected the respective structures.

The cave-in in the access tunnel which was 10 to 15 meters wide and about 9.0 meters beyond the level of the roof gave rise to an interesting system of interconnecting construction tunnels as shown in Fig. I. Transit Tunnel "A" which connects the tailrace and the access tunnels was utilized to excavate the access tunnel while the cave-in was being cleared and the necessary protective structure was being constructed. This same tunnel greatly facilitated the concreting of the tailrace tunnel and the excavation of the turbine vaults and draft tubes. This transit tunnel was later plugged with a concrete bulkhead provided with a door. Transit Tunnel "B" leads to the pilot tunnel in the upper bench heading of the power cavern. Transit Tunnel "D" made possible the excavation of the steep cable tunnel from below with the least interference to the excavation in the power station. Transit Tunnel "C" which is of coyote hole size was provided for pumpcrete lines and miscellaneous pipes and hoses needed in the construction. It is believed that this system of adits reduced materially the construction period.

The cave-in in the access tunnel and its consequences made a reliable comparison between the choice of a shaft or a tunnel as an access more difficult since the possibility of a cave-in over a wide area of such a complex geological formation can never be precluded. This gives rise to the question whether in areas of similar geology it is better to incline towards the choice of a shaft which traverses over a smaller area whenever the preliminary cost comparison does not indicate any appreciable difference.

Other difficulties were encountered during the construction which were successfully overcome by the contractors and the engineering staff of the National Power Corporation with the able help of the consulting engineers.

The author is to be complimented on a very comprehensive description of the power features of the Project. This project is the uppermost of six plants on the Agno River. The overall planning of the development of this river was

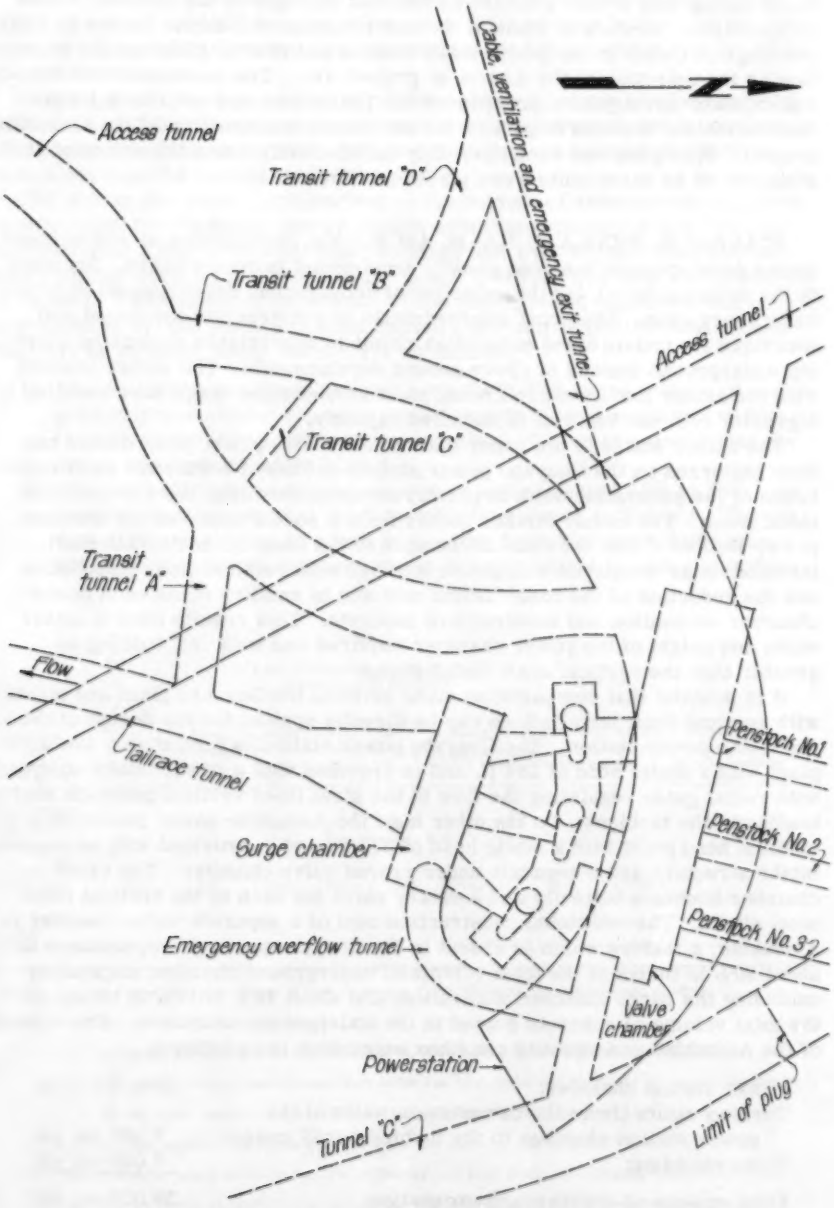

FIG. I PLAN SHOWING TRANSIT TUNNELS

made during this writer's tenure as General Manager of the National Power Corporation. Mention is made of the contributions of General Lucius D. Clay and Hugh J. Casey in the preliminary studies and overall planning and in particular the selection of the Ambuklao project site. The acceleration of the water power development program of the Philippines and indirectly its industrialization depends in part on the successful construction of the Ambuklao project. This plant has been operating satisfactorily since November of 1956 when two of its three units were placed on the line.

STAVROS N. NICOLAOU,[1] A. M. ASCE.—The construction of underground power developments has been greatly accelerated in recent years. Economy is the deciding factor for the selection of underground developments in practically every case. The great improvements in construction equipment and technique for underground excavation result in appreciable savings by adopting underground instead of above ground developments. The writer concurs with the author that an above ground plant at Ambuklao would have resulted in a greater cost per kilowatt of installed capacity.

The author states in his paper that the Ambuklao power plant design has been patterned on the Guayabo power station and that a horizontal shaft installation of the generating units is greatly advantageous over the vertical shaft installation. The author further quotes from a recent paper on the Guayabo power station(1) that the chief advantages of the Guayabo horizontal shaft installation as compared with power stations with vertical shaft installation are the reduction of the construction cost due to smaller volumes of power chamber excavation and substructure concrete. This results from a lesser width and height of the power chamber required and with unit spacing no greater than the vertical shaft installations.

It is doubtful that comparisons made between the Guayabo plant and plants with vertical shaft installations can be directly applied for the design of the Ambuklao power station. The Guayabo power station is a relatively low head plant with a static head of 184 ft. and is provided with a power intake equipped with radial gates regulating the flow to the steel lined vertical penstock shafts leading to the turbines. On the other hand the Ambuklao power station is a medium head plant with a static head of 572 ft, and is provided with an ungated intake structure and a separate under ground valve chamber. The valve chamber houses a butterfly and a rotary valve for each of the vertical lined steel shafts. The additional construction cost of a separate valve chamber in Ambuklao, a feature which is absent in the Guayabo power plant, amounts to about 20% in terms of the total volume of underground chamber excavation excluding the surge chamber excavation, and about 10% to 15% in terms of the total volume of concrete placed in the underground chambers. The volume of the Ambuklao underground chamber excavation is as follows:

Power station chamber:	27,000 cu. yd.
Turbine vaults (from the downstream walls of the power station chamber to the turbine scroll cases):	4,000 cu. yd.
Valve chamber:	8,000 cu. yd.
Total volume of underground excavation:	39,000 cu. yd.

1. Project Engr., Tippetts-Abbett-McCarthy-Stratton, New York, N. Y.

The 40% savings in favor of the Guayabo power station as quoted by the author in ref. (1) would decrease to less than 20% savings in favor of the horizontal shaft installations when adjustment is made for the volume of the valve chamber excavation.

Graphical comparisons, however, of the volume of power station chamber excavation of 150 underground power stations have indicated that horizontal installation for Francis turbines required slightly less volume of chamber excavation than the vertical installations.(2)

The writer has made a comparison of the volume of underground chamber excavation of the Ambuklao power station with three modern power stations equipped with Francis vertical shaft turbines and with similar head and capacity characteristics to the Ambuklao power station. The data are tabulated in Table I. The volume of excavation of the underground chambers has been expressed in cy. per kw of installed capacity per unit. Specific conclusions that may be deduced, from Table I are as follows:

(1) The Ambuklao power station required about 70% more volume of underground power station chamber excavation on a per kw and per unit basis than the Isere-Arc power station and about 40% more volume than the Binga and Santa Guistina plants.

(2) The Ambuklao power station spacing of units is about 15% to 50% greater than the vertical shaft installations shown in Table I. The effort of the Engineers to curve in plan the Ambuklao plant and arrange the units radially to converge to the tailrace tunnel in order to reduce

TABLE I

DATA ON UNDERGROUND POWER STATIONS COMPARED WITH AMBUKLAO

Country and Plant	Year of Operation	Plant Capacity M W	Turbine No. & Type (e)	Static Head Ft.	Unit Spacing Ft.	Power Station Dimensions-Ft. Length	Width	Height	Total Volume Excavation cu.yds.	Volume of Excavation cu.yds./Kw/Unit (d)
PHILIPPINES										
Ambuklao	1956	75	3Fh	572	49.5	230	42	66	39,000	0.130
Binga	1960	100	4Fv.	510	42.5	275	43	92	43,000	0.086
FRANCE (3)										
Isere-Arc	1954	135	4Fv.	500	33.0	357(a) 262(c)	55	90	70,000(b) 51,000(c)	0.076(c)
ITALY (4)										
Santa Giustina.	1951	108	3Fv.	560		200	78	110	40,000	0.093

Notes: (a) The power station length of the Isere-Arc Plant includes the transformer chamber.

(b) Including transformer chamber.

(c) Including generating hall and erection bay but excluding transformer chamber.

(d) The total volume of underground excavation has been converted to volume of excavation per kw of installed capacity and per unit basis by increasing the number of units by one to allow for the erection bay which occupies about the space of one unit.

(e) Number of turbines and type. Fv. = Francis vertical shaft, Fh = Francis horizontal shaft.

(f) For the vertical shaft installations the valves are housed in the power station chamber. For the Ambuklao power station the volume of excavation includes the power station, valve chamber and turbine vaults.

the volume of rock excavation is praiseworthy. However, field conditions such as excessive overbreak of the rock mass between the turbine vaults and draft tubes, and construction difficulties in excavation and concrete placement downstream of the power station chamber indicated the great advantages of a straight chamber in plan.

(3) The Ambuklao power station width is about the same as the Binga power station and about 25% less than the Isere-Arc power station. However both the Binga and Isere-Arc power stations do not extend downstream beyond the width indicated in Table I while the Ambuklao power station extends 28 ft. downstream of the 42 ft. width to provide the turbine vaults for the housing of the turbines.

The excavation of turbine vaults to house the turbines usually creates construction problems. Examples are the Ambuklao and Guayabo power stations both of which are familiar to the writer. Reference is made also to the authors Fig. 9, and 10. The rock mass between the turbine vaults was not sufficient to withstand even controlled blasting without excessive overbreak. The contractor removed the highly jointed and fractured rock mass by placing massive concrete blocks and columns between the turbine vaults. The excessive rock overbreak necessitated immediate support of the turbine vaults sides and roof arches to prevent the serious loss of the rock support around the downstream springing line of the power station roof arch.

(4) The arrangement of horizontal shaft installations such as the Ambuklao power station which are equipped with valves to regulate the flow through the vertical shaft penstocks require separate valve chambers. This involves not only additional cost for underground excavation and concrete placement but also construction difficulties and time delays in excavating a separate chamber above the power station chamber. In addition the special provisions required for access and ventilation of the valve chamber increase the overall construction cost. For the vertical shaft installations of Francis units and with heads of less than 700-750 ft. the valves are usually housed together with the generating equipment in the power station chamber. For higher heads it is preferable to house the valves in a separate chamber for safety reasons to ensure against flooding of the power station chamber in case of valve rupture due to high head.

As a general conclusion, it is believed that the final selection between underground horizontal shaft and vertical shaft Francis turbine installations should be determined in consideration of the merits of each method and in accordance with the specific conditions existing at each project. Knowledge of geologic conditions based on a complete and careful subsurface investigation supplemented with data on available construction equipment and construction techniques for underground excavation would greatly assist the design Engineer in determining the relative costs and making the final selection between horizontal and vertical shaft installations.

With reference to the author's discussion of inflow seepage into the access tunnel of the Ambuklao plant, the following data were furnished to the writer by the Resident Engineer during his inspection trip of the Ambuklao power station in October 1956: The seepage at the Ambuklao power station with the reservoir pool at El. 2430 (max. reservoir pool is set at El. 2467) exceeded the amount of 1500 gpm. The seepage water was flowing through the arch roof

and the weep holes of the concrete walls and the floor slab of the basement. The total seepage estimated including the inflow seepage from the access and cable tunnels exceeded 2,700 gpm. To decrease this seepage a grouting program was planned by grouting downstream of the power tunnel plug, in an upstream direction. The writer has been informed since that the inflow seepage has been somewhat reduced after completion of this grouting program.

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3. "The Isere-Arc Development," by C. Jaeger, Water Power, pp. 302 and 304, August 1953.
4. "Present Trends in the Design of Pressure Tunnels and Shafts for Underground Hydro-Electric Power Stations," by C. Jaeger, The Institution of Civil Engineering, London, 1955, pp. 162-165.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

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c. Discussion of several papers, grouped by divisions.

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